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SOME CONSIDERATIONS FOR THE DESIGN AND CONSTRUCTION OF IRRIGATION HEADWORKS IN SOUTH ARABIAN WADIS

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SOME CONSIDERATIONS FOR THE DESIGN AND CONSTRUCTION OF IRRIGATION HEADWORKS IN SOUTH ARABIAN WADIS

by

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The Aden Protectorate forms the hinterland of the Colony and Port of Aden situated on the south western corner of Arabia, and is bounded by the Yemen, the Rub el Khali desert of Saudi Arabia, Muscat and Oman, and the Gulf of Aden. It is an undeveloped area with a population of 660,000. The people largely depend for their livelihood on animal husbandry and agriculture, and have a depressed standard of living. The country is arid and mountainous, and of the total area of about 112,000 sq. miles, not more than 1% (700,000 acres) is cultivable. At present approximately 150,000 acres are cultivated , mainly in the larger wadis and their deltas, where layers of fertile alluvial silt have been deposited.

2. The catchments are rocky and steep and the annual rainfall, which may vary from 150-600 millimetres, is seasonal. This seasonal rainfall produces spate flows of short duration, and these are the main source of irrigation water. There is virtually no direct rainfall on the cultivable areas. There is a real need for more irrigated land on which a family can grow their own food and cultivate a cash crop. Cotton is the main cash crop and has become Aden's only agricultural export of value.

3. In this desert region, water is the vital necessity in any development aimed at improving living standards. Traditional irrigation methods are primitive but reasonably effective; however, in the particular geographic and economic circumstances there is an ever increasing demand for better irrigation facilities. At present the local farmers divert the spates on to their fields by building earth dams and deflectors across the wadis, but in high floods these are often washed away before the total acreage has been watered, and sometimes they cannot be rebuilt before the ensuing spates.

4. The territory relies largely on United Kingdom grants-in-aid to finance its development programmes, and with this assistance efforts are now being made to replace some of the more important traditional structures with permanent

works designed and constructed on accepted engineering principles. In order to do this certain basic data is required.

DATA FOR SCHEME EVALUATION

5. Before an administrative decision is taken to proceed with an irrigation project or work an engineering design and estimate together with an agricultural appreciation and an economic justification should be available. The engineering design and estimate would normally be based on preliminary investigations which should supply sufficient general information to enable a proper comparison to be made of the relative merits of competing schemes or works. The designs based on the preliminary investigation should not be considered as final, nor will the estimates for them be exact, but they must be adequate to obviate the need for extensive alterations later on if the scheme is accepted.

6. When the scheme is accepted further detailed surveys and investigations will be necessary before the final working drawings and estimates can be prepared. These will generally include the exact location of the irrigation structures and the alignment of all canals and watercourses, etc.

7. For the agricultural appreciation data will be required concerning the nature and extent of the irrigable area, soils and water classification, the crops to be grown and their projected cropping pattern, the anticipated rate of crop development, crop water requirements and expected yields etc. In addition, information concerning existing agriculture should be given including land ownership and size of farms; current land use and farm practices; the value, cost and net return of present crop production; crop marketing, processing and storage; and present land values.

8. An economic justification giving a preliminary estimate of the cost/benefit ratio could then be prepared from the information collected for a preliminary financing schedule and based on estimates of cost of water; estimates of annual revenue and costs; agricultural benefits, and estimates of increase in land values. Sometimes political factors will outweigh economic considerations when comparing the merits of competing schemes.

DATA FOR DESIGN

9. In the Aden Protectorate an important aspect of irrigation work is therefore the design and construction of permanent irrigation headworks in wadis to replace the temporary earth dams and deflectors built on traditional lines.

10. For this work an essential prerequisite is a large scale contoured map of the irrigable area with contour intervals of about 1m. With such a map, and knowing the cropping pattern and water requirements, the maximum discharge of the head regulator can be decided and the crest levels of the deflection weir and head regulators determined. In cases where contour maps are not available the existing irrigation system usually gives a reliable indication of what is required to decide the crest levels and discharge capacity of the

head regulators. Most of the cultivable areas in the Protectorate are covered by aerial photography and with a certain amount of ground control work and levelling sufficient information can be obtained for design purposes.

11. It is of fundamental importance to know the maximum flood flow that the weir will be required to pass. To assess the maximum flood flow with a reasonable degree of accuracy, the nature and extent of the catchment should be known and complete meteorological and hydrological records from suitable sites in the catchment should be available over a period of several years.

12. In the Protectorate the recorded information with regard to hydrology and meteorology is negligible and in order to assess the maximum flood on which the design should be based it is necessary to collect other information and to allow a larger 'factor of safety'. There is insufficient data even to calculate the average annual rainfall on the catchment area of any wadi, and an intensive questioning of the most responsible local inhabitants has to be carried out to ascertain the highest flood level within living memory and to have these high flood marks pointed out on the ground. By taking cross-sections to include the local high flood marks at the site of the structure, and again 1 mile upstream and 1 mile downstream, the high flood discharge may be calculated by determining the average cross-section and surface water slope using Kutters formula. To arrive at the design maximum flood it is advisable to increase the calculated high flood discharge by 20-30%.

13. Considerable savings in the initial cost of the structure and a greater degree of security may be achieved if adequate meteorological and hydrological information is available as a more accurate assessment of the maximum flood discharge and other flood characteristics can be made. Flood hydrographs have been prepared for the past few years from information recorded at the sites of a few recently constructed deflection weirs in various wadis, and these together with direct observation indicate that the flood characteristics in most of the wadis are similar and are of short duration (varying from an average of 5-20 hours) with a sharp peak. A knowledge of the flood characteristics is important when making provision for adequate structural safety, particularly with regard to the energy dissipators, freeboard etc.

HYDRAULIC DESIGN FACTORS

14. Having decided on the design maximum flood it is possible to calculate the maximum flood level by using the standard broad crested weir formula:-

$$Q = c\ B\ H^{3/2}$$

Rearranging: $H = \left(\frac{Q}{cB}\right)^{\frac{2}{3}}$ = total energy head above crest.

The head due to the velocity of approach $h_v = \frac{v^2}{2g}$ must be deducted from the total energy head (H) to get the maximum water level above the crest. An allowance for freeboard of 2 ft. to 3 ft. should be added to this level to obtain the top level of abutments, piers, flank walls, etc.

15. The minimum water level will be the weir crest level for a non-regulating

deflection weir. This information is required for computations of hydraulic gradients through the structure.

16. It is also necessary to know the river bed levels at the weir site and these can readily be obtained from the cross-sections. The depth of the average bed level below the crest of the weir will then be known.

17. The following basic factors necessary for the design of the weir in relation to surface and sub-surface flow must be decided.

Type of weir and canal headworks.
Afflux.
Width of waterway.
Discharge per foot run.
Crest levels.
'Silt factor' (f) of the bed material.

(a) Type of Weir and Canal Headworks.

Spate spreading in Aden is mostly carried out by deflection weirs which do not have control gates. These non-regulating weirs are often designed as broad crested weirs on permeable foundations. A canal head regulator incorporating a silt sluiceway is usually sited on each flank of the weir. The sluiceway serves to reduce the amount of stream bed load entering the canal head regulator and assists in maintaining a channel to it. However, the sluiceway gives little or no protection against the silt carried in suspension. This suspended silt will enter the canal and must be removed by silt ejectors downstream of the headworks. The sluiceway is generally placed normal to the axis of the weir so that its discharge will be parallel to the weir flow and in order that the canal head regulator should function effectively, it is set at an angle to the sluiceway. In the Aden Protectorate an angle of 30° between the axis of the sluiceway and the face of the canal head regulator has been used. This was a satisfactory choice for the particular features of the sites but this angle may be varied from almost 0 to 90 degrees and may best be determined by model studies if the magnitude of the work warrants the expenditure. The cills of the canal head regulator and sand sluice are generally about 1.0 ft. and 3.0 ft. respectively below the crest of the weir. With this arrangement and by providing a smooth pavement in the approach channel and a divide wall between the weir and the silt sluiceway, its scouring capacity is increased, and the flow of heavy sand in the lower layers is concentrated towards the sluiceway so that only the finer silt in the upper layers is admitted into the canal. Control of the sluiceway is sometimes effected by using timber 'needles', mainly on the grounds of economy, rather than a control gate.

(b) Afflux.

It is necessary to prepare a chart showing the headwater and tailwater rating curves. For broad crested weirs the headwater levels are decided as explained in para14. The tailwater levels can be calculated by taking cross-sections for some distance downstream of the structure, or until some permanent obstruction or restriction is encountered in the bed of the wadi. The tailwater depth at the weir can be ascertained by computing a backwater curve from the point of restriction, or by using any of the accepted open channel flow formulae. The difference between the headwater and tailwater rating curves (afflux) is then known. The afflux becomes very small when large spate flows

pass over low deflection weirs, a condition often encountered in Aden wadis, and the changes in afflux must be allowed for in the design of the weir. Whereas large flows may pass over the weir safely, detailed analysis will indicate that it is often the smaller discharges which are likely to do the damage and their size will become an important criterion in deciding the dimensions of the structure.

(c) Width of Waterway.

The length of the weir between the abutments of the canal head regulators, excluding the width of any piers, will give the effective width of the waterway and will largely be determined by the physical features of the site, but this width should compare favourably with the minimum stable width for the maximum flood discharge given by the Lacey formula:- $P_w = 2.67\ Q^{\frac{1}{2}}$, where P_w = minimum stable width in feet; and Q = maximum flood discharge in cusecs.

(d) Discharge per foot run.

In India for weirs founded on alluvial sands or shingle the safe rule has been to provide sufficient waterway to pass the maximum flood discharge with an afflux not exceeding 3 ft. The smaller the design maximum discharge per foot run, the smaller will be the dynamic action of the water and the depth of scour; the length and thickness of the pervious and impervious floors will also be less. The cost of providing a longer weir with a lighter cross-section must be compared with that of a shorter weir with a heavier cross-section.

(e) Crest Levels.

The crest level of the weir is related, as stated in para.17(a), to the cill level of the canal head regulator which in turn is established from the topographical features of the irrigable area to be commanded.

(f) Silt Factor 'f' of the bed material.

In order to determine the depth of scour likely to occur upstream and downstream of the weir during a maximum flood, it is necessary to know not only the maximum discharge per foot run, but also Lacey's[1] silt factor (f) for the wadi bed material at the weir site. The silt factor is related to the mean particle diameter (m) of the bed material by the formula $f = 1.76m^{\frac{1}{2}}$ where 'm' is in mm. The silt factor can therefore be computed after carrying out a sieve analysis on a representative sample of the bed material from the site of the weir to determine the mean particle diameter in millimetres.

DESIGN OF WEIR FOR SURFACE FLOW

18. To design a non-regulating broad crested weir on a permeable foundation, a series of calculations will have to be made for the various conditions of discharge in order to determine:-

The length of crest.
The position and details of the standing wave.
The dimensions of the energy dissipator and appurtenances.
The depth of scour in relation to the depth of sheet piling, and length of flexible apron.

19. An example of the procedure used in designing the Bateis Weir, Wadi Hajr, may serve to indicate how these details may be determined.

For design maximum discharge:-

Q = 58000 cusecs.
B = 650 ft. (length of waterway)
L = 1 ft. (afflux)
Crest level = 37.20
Slope of glacis (2) 1 in 5

(a) To determine upstream water level:

Discharge/ft.run $q = \frac{Q}{B} = \frac{58000}{650} = 89$ cusecs

Value of c = 3.1 in formula $Q = cBH^{3/2}$
where c = coeff. of discharge
and H = total energy head above crest (ft.)
The value of c increases from about 3.0 to 3.1 as the intensity of discharge increases (3).

$H = \left(\frac{q}{c}\right)^{\frac{2}{3}} = \left(\frac{89}{3.1}\right)^{\frac{2}{3}} = 9.30$ ft.

Velocity of approach $v = \frac{q}{H} = \frac{89}{9.3} = 9.6$ ft./sec.

Velocity head $h_v = \frac{v^2}{2g} = \frac{92}{64} = 1.44$ ft.

Therefore depth on crest $= H - h_v = 7.86$ ft.

Upstream water level $= 37.20 + 7.86 = 45.06$.

(b) To determine length of crest:

Let length of crest = 2H = 2 x 9.30 = 18.60

(c) To determine position and details of standing wave:

Slope of glacis 1 in 5
Critical depth $C = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} = 6.2$ ft.

Afflux L = 1 ft. $\frac{L}{C} = \frac{1}{6.2} = 0.162$

Reference Plate V in Vol.III of "Irrigation Engineering" by K.R.Sharma -

For $\frac{L}{C} = 0.162$; $\frac{K+F}{C} = 1.870$; $\frac{x}{c} = 0.635$; $\frac{Y}{c} = 1.485$

where K = (H) Total upstream energy head
F = Elevation of crest above point of formation of standing wave
L = Loss in head in standing wave
x = Depth of flow upstream of wave
y = Depth of flow downstream of wave.

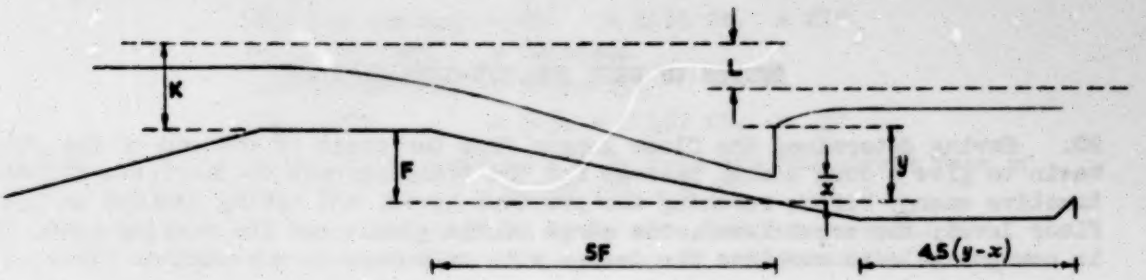

Figure I

Therefore F = 1.870 C-K = 1.870 x 6.2 - 9.3 = 2.3

and jump will occur 5 x 2.3 = 11.5 ft. horizontally from top of glacis.

x = 0.635C = 3.94 ft. y = 1.485C = 8.60 ft.

Downstream water level = 45.06 - 1.00 = 44.06

Jump height elevation for this discharge:

= crest elevation - F + y = 37.20 - 2.30 + 8.60 = 43.50

Min. floor level = Downstream water level - y = 44.06 - 8.60 = 35.46

The above calculations are repeated for various discharges throughout the range and a design adopted which satisfies all flow conditions.

(d) To determine the dimensions of the energy dissipator and appurtenances:

(i) The following table prepared for the Bateis Weir shows the length of the stilling basin[4] required for various discharges under the set of hydraulic conditions for which the structure was designed (See Figure I).

Discharge in cusecs.	x ft.	y ft.	y-x ft.	4.5 (y-x) ft.	5F ft.	5F + 4.5 (y-x) ft.	
58,000	3.94	8.60	4.66	21.0	11.50	32.50	
45,000	2.82	9.20	6.38	28.6	23.95	52.55	
30,000	1.74	7.74	6.00	27.0	31.20	58.20	
14,500	0.87	5.62	4.75	21.4	37.50	58.90	(60' (provided
5,000	0.30	3.32	3.02	13.6	41.25	54.85	

It will be seen that about 60 ft. is required horizontally from the top of the glacis to the end of the stilling basin and as the glacis is 40 ft. long, a stilling basin length of 20 ft. was adopted.

(ii) The significant discharge on which the design of the stilling basin appurtenances should be based would appear to be about 14,500 cusecs and from Elevatorski's recommended dimensions[4] the following figures were obtained:

For Q = 14,500 cusecs; x = 0.87 and y = 5.62

Chute Blocks: Block height = x = 0.87 ft. = 10"
Width of block = $\frac{3}{4}$x = 0.65 ft. = 8"
Spacing of block = $\frac{3}{4}$x = 0.65 ft. = 8"

DESIGN OF WEIR FOR SUB-SURFACE FLOW

20. Having determined the floor length from the crest to the end of the stilling basin to give a long enough passage for the standing wave to dissipate its destructive energy before reaching the pervious apron, and having decided on the floor level, the crest level, the slope of the glacis and the working head, it is now possible to complete the design with reference to sub-surface flow.

21. Two essentials remain to be considered for the weir design, namely: the uplift pressures on the weir floor and the exit gradient. These two essentials are inter-connected. For any given foundation profile of a weir in a given class of soil, there will be a definite distribution of pressure and a definite exit gradient. To safeguard against undermining, the exit gradient must not be allowed to exceed a certain safe limit, generally 1/5 to 1/7. The uplift pressures must be kept as low as possible, consistent with safety at exit, so as to keep floor thickness to a minimum.

22. From practical considerations and particularly to avoid a costly dewatering operation it was necessary to restrict the depth of the downstream curtain wall to about 6 ft. This cut off depth together with an acceptable exit gradient determined the overall length of the impervious apron. The exit gradient was checked for various lengths of impervious apron under the worst hydraulic condition i.e. the 'no flow' condition with water at crest level and the wadi dry downstream. The head across the structure was then 9 ft. By these trial and error methods a suitable length of impervious apron was found to be 100 ft. Plate No.XLVIII in "Handbook of Instructions for the Design of Lined and Unlined Channels and Masonry Works", Bhakra Nangal Project(5), is a graph for determining exit gradients. Referring to this chart for $\frac{H}{d} = \frac{9}{6} = 1.5$ and $\frac{b}{d} = \frac{100}{6} = 16.7$ then $G_e = 0.162 = \frac{1}{6.2}$. This factor of safety (6.2) is within acceptable limits.

23. It now remains to determine the thickness of the impervious apron with reference to uplift pressures and the standing wave. The floor may be designed as a purely gravity section at each point to withstand the indicated pressure at the various points acting on the entire base area. The tension of the floor concrete or masonry will be an additional safeguard and afford a useful factor of safety.

24. The maximum uplift for which the floor will have to be designed will be affected by the standing wave trough and in determining the thickness of the floor for the trough the following points should be taken into consideration. The actual position of the standing wave occurs some distance (y-x ft.) upstream of the calculated position and therefore the actual depth of trough will be somewhat smaller than calculated. The uplift pressure due to the maximum depth at the trough should not be treated as a point load since the floor slab has always got some beam action and may be designed in sections for the average uplift over each section. The maximum pressure may be taken as three quarters of the actual maximum to allow for this distribution of pressure. Due to the downward force exerted on the floor by the jet of water flowing down the glacis, which is equal to the rate of change of momentum in the vertical direction, the thickness of the floor in the vicinity of the toe may be reduced.

Baffle Piers:	**Height**	**= 2.5x = 2.19 ft.**	**= 26"**
	Width & spacing	**= 2x = 1.75 ft.**	**= 21"**
	Upstream face	**= Vertical**	
	Downstream face	**= Slope 1:1**	
	Top width	**= 0.5x = 0.437 ft.**	**= 5"**

The baffles will be located at a distance = 0.9y from the downstream face of the chute blocks: 0.9y = 0.9 x 5.62 = 5.058 ft. = 5 ft.

These baffle piers will have to withstand heavy abrasion due to running boulders and shingle in water, and it is necessary to reinforce the upstream face and corners to meet this attack.

End Cill: Height = 0.2y = 0.2 x 5.62 = 1.12 ft.
Horizontal top width = 0.04y = 0.04 x 5.62 = 0.225 ft.
Upstream face - Slope 1:2. Downstream face - Vertical.

(e) To determine depth of scour in relation to depth of sheet piling and length of flexible apron.

Q = 58000 cusecs. q = 89 cusecs.

Assuming crest level as zero datum:

Downstream Floor level	**= - 8.0**
Upstream H.F.L.	**= + 7.86**
Downstream H.F.L.	**= + 6.86**

The bed material at the site varies from coarse sand to shingle and boulders. The Lacey silt factor (f) of the bed material was 7.1 and was determined as described in para.17(f).

For a regime flow the value of the hydraulic mean depth "R" in terms of "q" and "f" is given by the following Lacey equation:-

$$R = 0.9 \frac{(q^2)^{\frac{1}{3}}}{(f)} = 0.9 \frac{(89^2)^{\frac{1}{3}}}{(7.1)} = 9.36 \text{ ft.}$$

Depths of scour to be guarded against, in terms of "R", are specified on page 152 of C.B.I. Publication (2) No.12. For the design of sheet piles it will be enough to take the piles to a depth of scour equal to "R". For the design of flexible aprons upstream and downstream of the weir, the depth of scour to be guarded against should be 1.5R and 2R respectively to suit local concentrations of flow. These scour depths are measured below the maximum flood levels. In the design of the sheet piling the significant factor was the exit gradient and not the depth of scour.

For the design of flexible aprons:

Upstream depth of scour	**= 1.5 x 9.36 = 14 ft.**
Downstream depth of scour	**= 2 x 9.36 = 19 ft.**

By providing a sloping floor upstream taken down 6 ft. below the crest of the weir and ending with an 18 inch deep toe wall, the scour depth of 14 ft. was guarded against and no flexible protection was considered necessary upstream.

On the downstream side the scour depth of 19 ft. was 4 ft. below the floor level of the stilling basin. To provide flexible protection against this scour, at a slope of 1:2, a length of apron of approximately 9 ft. is required and it was decided to use stone filled gabions 10 ft. long x 1.5 ft. deep.

25. The position of the standing wave, with water profile and hydraulic gradient lines for various stages of flow, as previously calculated, can be superimposed on a cross-section of the weir and a graphical solution for the maximum uplift for which the floor must be designed can be obtained from the resulting diagram. The thickness of the floor may be made equal to three quarters of the maximum differential head of water in feet and with a weight ratio of 62 to 88 (water to mass concrete in water) this provides a factor of safety of 1.4.

CONSTRUCTION

26. Generally speaking modern construction techniques have so far been little used in the Aden Protectorate but the people are very skilled at making the fullest use of local materials and the quality of masonry work in particular is very high. Bearing this in mind and taking into account the shortage of skilled supervisory staff, structural design must incorporate as far as possible masonry and mass concrete work and avoid wherever possible reinforced or pre-stressed concrete. Although the standard of the masonry is high, the quality of the mass concrete is often low as grading of the aggregates is done in many cases by eye and general site control is poor. For these reasons, relatively low design stresses (500 lbs/sq.in.) for concrete are used. Sites are in many instances not easily accessible, communications are poor and the cost of transport of imported materials and equipment is therefore very high. Nevertheless, as relatively cheap labour is available and with the maximum utilisation of local materials, works can be executed at very reasonable costs and the general economy of the country benefits by reducing to a minimum the use of imported materials and plant.

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